CONCRETE AND CONSTRUCTIONAL ENGINEERING

INCLUDING PRESTRESSED CONCRETE

MARCH 1959 **VOL. LIV. NO. 3**

FIFTY-FOURTH YEAR OF PUBLICATION

PRICE 2s. ANNUAL SUBSCRIPTION 24s. POST FREE. $3.90 in Canada and U.S.A.

LEADING CONTENTS

No. 616 ISSUED MONTHLY *Registered for Canadian Magazine Post*

INDEX OF ADVERTISERS

V

In
ye
ti
"
cr
in
re
O
re
in
go
ar
an
Re
us
an
wi
th
of
me
cr

for
cer
cos
ho
co
ext
fro
an
flo
ma

C

C

Volum

In the
years a
tion, c
"scien
crete i
in imp
refined
Opinio
remain
involve
good j
are em
and, a
Resear
use th
and ex
with th
that w
of the
ment t
creting

In
for 10
cemen
costing
howev
could
extra
from t
and sk
flow b
materi

M

CONCRETE
AND
CONSTRUCTIONAL ENGINEERING
INCLUDING PRESTRESSED CONCRETE

Volume LIV, No. 3. LONDON, MARCH, 1959.

EDITORIAL NOTES

The Theory and Practice of Making Concrete.

In the introduction to " Basic Principles of Concrete Making ", published thirty years ago by the Director of Research of the American Portland Cement Association, complaint is made that the replacement of " rule of thumb methods " by " scientific riddles " in the production of concrete mixtures was resulting in concrete inferior to the 1 : 2 : 4 mixtures. Examples are given of bad concrete in important structures " built with the aid of what was believed to be the most refined scientific effort to produce a mixture of ideal excellence and economy ". Opinions are expressed that scientific refinements had bedevilled an art that remained intensely practical, and that if good concrete was obtained in spite of involved methods of proportioning it was the result only of good fortune and the good judgment of the engineer on the site. To-day more " scientists " than ever are employed in laboratories on what they ungrammatically call " mix design " and, according to a report* issued by the Department of Scientific and Industrial Research, some of them are still bedevilling the men on the site who have to use the specifications they prepare. The report clearly shows the difficulties and extra cost that may be caused by specifications that are concerned only with the production of concrete containing the least amount of cement and water that will give the required strength; if specifications for concrete take no account of the cost of compacting it in narrow shutters or around closely-spaced reinforcement they will be expensive luxuries that should have no place in practical concreting.

In one of the cases quoted it was claimed that a laboratory-made specification for 1000 cu. yd. of concrete in a framed structure would save 10 per cent. of the cement, valued at £165. This worked out at only 3*s.* per cubic yard of concrete costing 320*s.* per cubic yard when compacted in the shutters. It was found, however, that the concrete was so stiff that its flow from the skip to the shutters could not be regulated, with the result that it had to be tipped on to a banker and extra labour, costing more than 3*s.* a cubic yard, was employed in barrowing it from the banker and shovelling it into the shutters. Indeed the use of the crane and skip could not be used for part of the work because the concrete would not flow by gravity, and a rope and pail were resorted to for hoisting and placing the material. The exact cost of the extra labour employed and of the time wasted at

* An abstract of this report is given on page 125.

and behind the mixer could not be ascertained, but it is thought that it may have cost £500 in wages to save cement worth £165.

Another example described is an eight-story building containing about 8000 cu. yd. of concrete mixed in accordance with a specification that was calculated to save cement valued at £1600. This was a framed structure in which each upper story was the same as that below and rapid construction should have been possible. Here again the mixture was so stiff that its flow from the skip could not be regulated, and in this case also it had to be tipped on to a banker, barrowed to where it was required, and shovelled into the shutters. So stiff was this mixture that at times an immersion vibrator was used to empty the skip; although the consistence varied, some assistance was always necessary to empty the skip. Even when concreting floor slabs extra labour had to be employed to empty the skip and to spread and compact the mixture. In this case also the extra cost of labour more than outweighed the value of the cement saved, and the same result was seen on other sites where similar efforts had been made to save cement by the use of stiff mixtures. The extra cost of a slower rate of the actual operation of placing concrete can be ascertained fairly exactly, but the cost of the slower rate of mixing and the delay of other trades cannot be known.

A comparison is made of a framed structure containing more than 30,000 cu. yd. of concrete, and where every effort was made to hasten construction. The mixtures were in nominal proportions for different parts of the building, and the consistence was left to the discretion of the engineers on the site. As a result the workability of the concrete was such that it flowed into the shutters directly from the skip with little assistance and without segregation, and concrete was placed in wall shutters at the rate of 0·6 man-hour per cubic yard.

In theory the obvious way to cheapen concrete while maintaining its compressive strength is to reduce the content of water and cement, but the stiff mixtures that result have limited applications in practical work. In some cases, such as the production of precast concrete in a factory, stiff mixtures can be economical because means are available of transporting and tipping stiff concrete into the moulds, compaction by vibration is cheaper and can be better controlled, and the higher early strength of stiff mixtures enables the moulds or prestressing beds to be re-used sooner. In the case of thin walls, columns, beams, and other members cast in place, however, the ideal economical concrete of the theorist can result only in delaying construction and increasing costs out of all proportion to the value of the cement saved by the use of leaner and stiffer mixtures. It can also give a sense of frustration to the engineer on the site when he finds that the mixtures specified are unsuitable for their intended purpose and can be placed only at excessive cost. On a recent contract in London ready-mixed concrete supplied to a rigid "economical high strength" specification could be used in ribbed floors only after it had been tipped on to a banker and water added until it was sufficiently workable for the purpose. Laboratories have their uses in selecting gradings and proportions that will result in maximum density and workability with a low water-cement ratio, but it should then be left to the engineer on the site to vary the quantities of cement and water in the specified ratio to produce concrete that is sufficiently workable for its purpose. It is useless to train and employ experienced supervisors if they are forced to follow a laboratory specification that results in poor or unnecessarily expensive concrete.

Tests on Concrete Tunnel-linings.

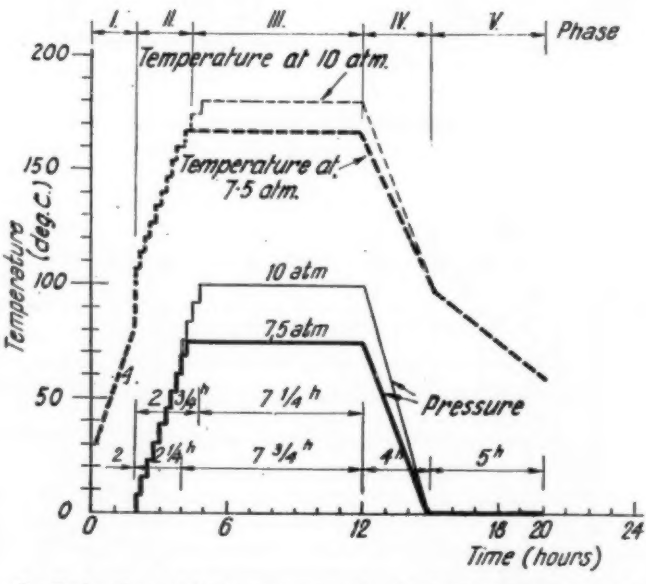
High-quality Concrete.

By PROFESSOR DR. CH. SZÉCHY (Budapest).

The principal requirements for segmental tunnel linings are (1) Ability to support the external and internal loads with the smallest possible weight; (2) Watertightness of the segments and the joints between them against pressure heads of 5 to 8 atmospheres; (3) Sufficient strength and accuracy of the contact surfaces to resist the thrust of the jacks propelling the tunnelling shield; (4) Sufficient resistance to corrosion and chemical attack; (5) Weights and dimensions such that there is little likelihood of damage during delivery and erection, and ease of handling.

The purpose of the experiments described in the following was the development of a reinforced concrete segment which would meet all these requirements. The first task was the production of watertight concrete with (a) minimum strengths of 6430 lb. per square inch at 24 hours and 8570 lb. per square inch at 28 days (measured on 8-in. cubes); (b) minimum tensile strengths of 570 lb. per square inch at 24 hours and 786 lb. per square inch at 28 days measured on prisms 6 in. square and 2 ft. 7 in. long; and (c) negligible percolation through a thickness of $3\frac{3}{16}$ in. of concrete subjected to a pressure-head of 10 atmospheres.

Attempts were made, without success, to produce such concrete by steam-curing at a temperature of 85 deg. C. at normal atmospheric pressure. Concrete



I.—Preheating without pressure. II.—Increased temperature and pressure. III.—Steaming under pressure. IV.—Reduced temperature and pressure. V.—Cooling in Steam-Chamber.

Fig. 1.—Steam-curing Process.

produced in this manner was sufficiently strong but insufficiently watertight. The use of steam curing at a temperature of 167 deg. C. and a pressure of 7½ or 10 atmospheres, in cylindrical kilns, was, however, completely successful. The five phases of the process are shown in *Fig.* 1. Segments made in this way have entirely fulfilled all the conditions. They were completely watertight against a pressure-head of 14 atmospheres. They were also highly resistant to chemical attack; for example, when specimens cured in air and in the manner described in the foregoing were stored in a strong solution of magnesium sulphate, only the specimens cured in air were attacked. Changes in the fineness modulus or grading of the aggregates and a pronounced reduction of the water-cement ratio had little effect on the properties of concrete cured in this way. A cement content of 590 lb. per cubic yard was sufficient to provide the prescribed strength. The aggregate was obtained from the River Danube, and consisted of equal quantities

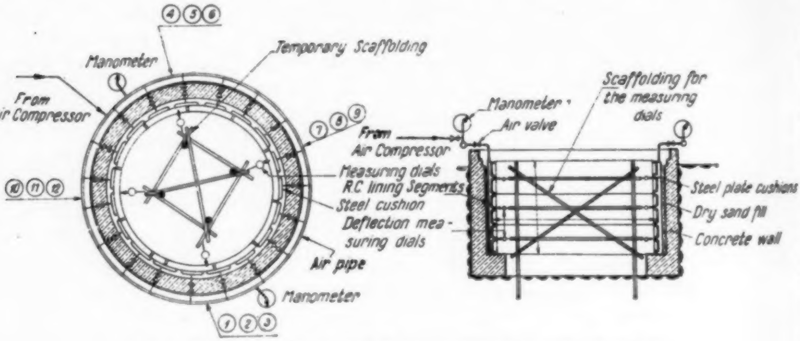


Fig. 2.—Arrangement of Transverse Loading Test.

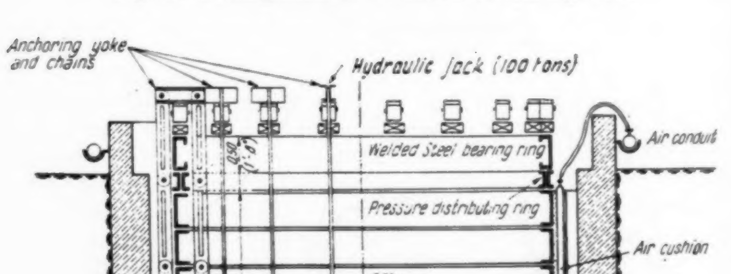


Fig. 3.—Arrangement for Longitudinal Loading Test.

Fig. 4.—Steel Mould on Vibrating Table.

of sand with a maximum size of $\frac{3}{16}$ in. and gravel with a maximum size of $\frac{3}{8}$ in. The water-cement ratio was between 0·37 and 0·40. The addition of powdered quartz increased the strength, as also did natural fine sand but to a slightly smaller degree. The shrinkage of the concrete was negligible and its adhesion to the reinforcement was normal.

Tests were then made to ascertain the strength of the segments under conditions similar to those in a tunnel. They served also to develop the most effective structural arrangement and reinforcement of the segments. The segments were tested in complete rings in order to determine their resistance to transverse and longitudinal stresses, and then separately to find the best arrangement of the ribs, reinforcement, and joints.

The rings were tested in a vertical circular well with an inside diameter of 23 ft. 3 in. and 16 ft. 8 in. deep in which six complete rings could be placed one above the other. The transverse load was applied by means of steel cushions on the outside of the rings. The pressure was exerted by forcing compressed air into the cushions, which were embedded in a thin layer of dry sand (*Fig.* 2) to ensure that the load was evenly distributed. Axial loads were applied by means of eighteen hydraulic jacks, each of 100 tons capacity, arranged as shown in *Fig.* 3. Paper strips were inserted between the rings to reduce the effect of surface irregularities.

Satisfactory results were obtained from all the tests. The deflections produced by the transverse load were much smaller than those calculated, particularly when the load was not uniform; in this case residual deformations were observed, the joints permitting the ring to adjust its shape to suit the line of thrust, thereby reducing the bending moments. Surface irregularities on segments made in timber moulds caused some spalling of concrete under axial load, and some minor spalling occurred in segments cast in the steel moulds shown in *Fig.* 4, indicating that the method of compaction and the contact surfaces of the moulds should be improved.

Single segments were tested in bending only, compression only, and bending combined with compression. The tests led to improvements in the arrangement of the reinforcement, and demonstrated the importance of stirrups and bent-up bars. The flanges were found to require reinforcement to resist the combined action of shear and bending, and two-way reinforcement between the ribs proved to be most efficient. Hooks and bent bars were found to be necessary around the bolt-holes and injection-holes to distribute the high local stresses.

Two types of segment were tested, one about 1 ft. 8 in. wide with two internal ribs and the other about 3 ft. 4 in. wide with three internal ribs (*Figs.* 5 and 6). The results showed that the larger units were about twice as strong as the smaller, although they included only about 50 per cent. more material; the number of

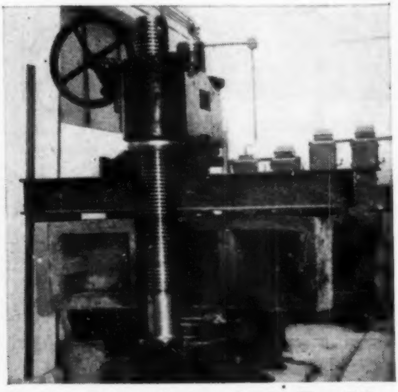


Fig. 5.—Test on Segment with Two Ribs.

Fig. 6.—Test on Segment with Three Ribs.

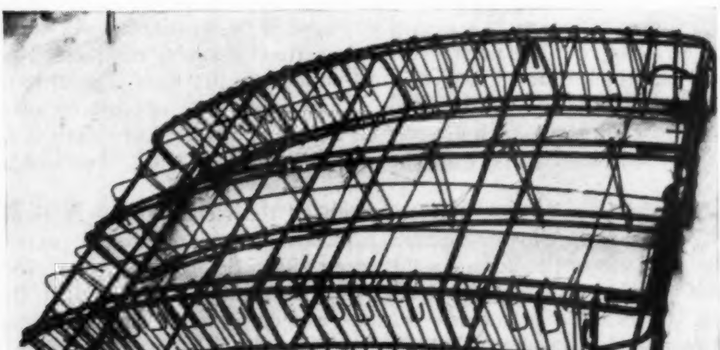


Fig. 7.—Reinforcement for Segment with Three Ribs.

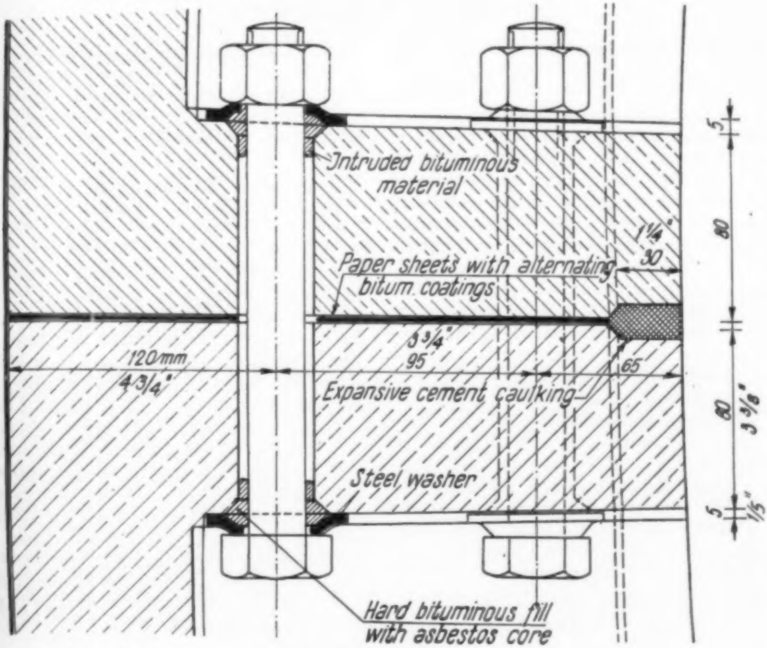


Fig. 8.—Details of Joint.

joints in a tunnel would also be halved if the larger units were used, and they were therefore recommended. The reinforcement for a large unit is shown in *Fig.* 7.

Further tests showed that the most efficient arrangement of bolts is that shown in *Figs.* 6 and 8, and that the most suitable jointing material consists of four layers of paper with layers of bitumen between them. Joints formed in this way resisted a water pressure of 8 atmospheres. Grooves about $\frac{3}{4}$ in. wide and $1\frac{1}{4}$ in. deep were also formed between the outer ribs (*Fig.* 8); these were caulked with expansive cement.

It is estimated that the cost of reinforced concrete segments made in this way would be between 75 and 80 per cent. of that of cast iron segments and that the saving in ferrous metal would be more than 90 per cent.

The tests were made and the results evaluated by Mr. J. Illéssy, under the direction and direct supervision of Professor L. Palotás and on the initiation and under the partial direction of the writer.

Book Reviews.

"Elementary Reinforced Concrete Design." By W. Morgan. (London: 1958. Edward Arnold (Publishers), Ltd. Price 28s.)

THIS is the second edition of a clearly written and concise book. It has been brought up to date in accordance with B.S. Code No. 114 (1957), and chapters have been added on retaining walls and prestressed concrete; there is also new matter on beams of non-rectangular cross section. It is a book which should continue to prove useful to students of architecture, building, and surveying as the first two chapters deal with the principles of the theory of structures and the calculation of bending moments and shearing forces, thus providing in one volume much of the information required for their examinations. A commendable feature is the clarity of the diagrams. Many engineers would, however, prefer to use wider beams than those shown in some of the examples, thus reducing the number of layers of bars and allowing easier placing of concrete.

"The Industrial Cooling Tower." By K. K. McKelvey and Maxey Brooke. (London: D. Van Nostrand Co., Ltd. 1959. Price 90s.)

A BRITISH civil engineer and an American chemical engineer have collaborated in the preparation of this book of more than 400 pages. Information abstracted from technical literature is brought together to give a comprehensive account of the practice of cooling water at industrial works and power stations. Most of the book deals with thermal problems and general design as regards dimensions of cooling towers constructed mainly of wood or concrete. In a short chapter on structural design, the stresses in a hyperboloidal concrete shell are analysed, and a complete numerical example (in the metric system) is given of the design of a natural-draught concrete tower partly of precast construction. In a chapter on construction, details of scaffolding and shuttering for towers of several shapes and for various methods of concreting are given with a completeness that is generally lacking in books devoted mainly to the design of plant from an operational point of view. The heading of a short section, "Concrete Mix Design", is misleading as much of this deals with temperature and other effects. A questionable recommendation is that concrete with a small cement content and low water-cement ratio should be specified so that the contractor will have to apply considerable effort to compact it.

"Zehnteilige Einflusslinen für durchlaufende Träger." Volume II. Seventh edition. By G. Anger. (Berlin: Wilhelm Ernst & Sohn. Price 41 D.M.)

TABLES are given of bending moments, shearing forces, and reactions for continuous beams of up to five spans. Many ratios of spans are included, and both free and fixed-end supports are considered. Data are given for uniform loading, and factors are introduced for any other type of load; reference must be made to Volume I for information concerning these. In this edition the ratios of lengths of spans are given in increments of 1 per cent., thereby avoiding the need for interpolation. Even with this improvement, however, the analysis of beams with complicated loads by means of the tables is so complex that it may be quicker, and safer, to work from first principles.

"Bewegungsfugen im Beton und Stahlbetonbau." By A. Kleinlogel. (Berlin: Wilhelm Ernst & Sohn. Sixth edition. 1958. 272 pages. Price 34 D.M.)

As in the previous edition, some abstracts from which were given in this journal for September, 1955, upwards of 300 expansion and other joints suitable for various types of plain and reinforced concrete structures are described and illustrated. Diagrams showing correct and incorrect methods of forming joints are given.

In this edition new types of joints suitable for a tank or swimming pool are illustrated. Joints in the cantilevered wall of such a structure are provided by leaving a gap 10 in. wide at intervals of 40 ft. to 53 ft. A ⅜-in. layer of plastic bitumen is applied to the rebated faces of the wall, and a strip of jute followed by a ½-in. layer of bitumen is then applied. The gap is closed with 1 : 8 concrete about ten days before the tank is filled, the concreting being done at a low temperature (for example, at night). Four vertical bars retained by binders are provided in the gap.

Prestressed Footbridges.

Bridge in Wales.

A PRESTRESSED footbridge (*Fig.* 1) of 108 ft. span over the River Teifi at Abercych, near Cardigan, replaces a steel structure. The bridge, which is 4 ft. wide, is a simple tee-beam 3 ft. 6 in. deep prestressed with seven 12-wire cables. A camber of 9 in. is provided, which enables the cables to be as straight as possible thereby reducing friction during tensioning. Five of the cables are inclined gently upwards towards the supports (*Fig.* 2). Stiffeners are provided at centres of 9 ft. 8 in. and these also support the posts for the handrails. The beam comprises separate precast tee-blocks, the weight of each of which was restricted to 10 cwt. to suit the lifting gear available; the two end anchor-blocks are heavier. The beam is carried on a simple concrete rocker bearing in a recess at each support. After tensioning the cables one rocker was grouted in its pocket to form a fixed-end bearing and

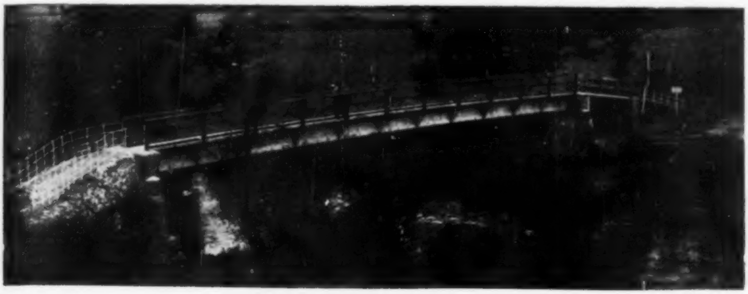


Fig. 1.—Bridge in Wales.

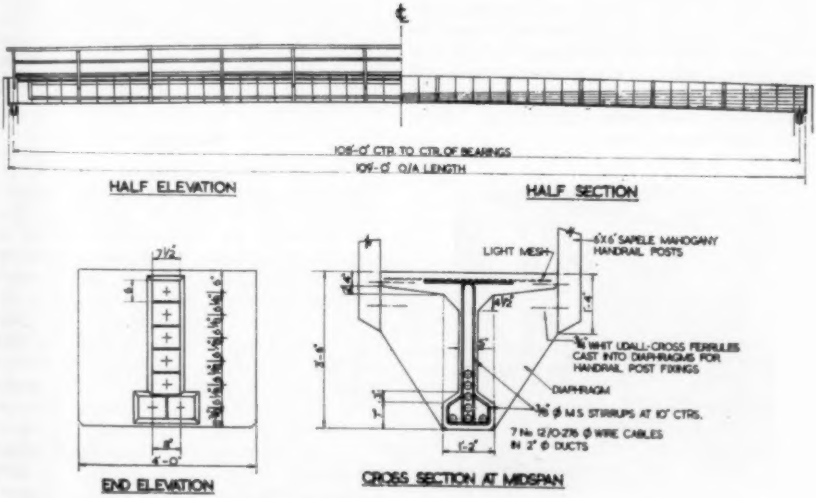


Fig. 2. Details of Beam.

the other was embedded in bitumen to allow for movement. The bridge is approached from one side by a short ramp of prestressed planks spanning over a flood-relief channel.

Construction.

A staging 12 ft. wide was erected to provide a temporary passage for pedestrians and to serve as a working platform during the demolition of the old structure and the construction of the new bridge. Because access to the site was difficult, the precast blocks were delivered to a convenient position beside the road from where they were taken in the required order by a 1-ton mobile crane to the beginning of the footpath. Here they were placed on a hand-trolley (*Fig.* 3) which, under the control of a hand-winch, was guided down a steep slope to the staging. The blocks were placed in position by a simple travelling gantry with a slow-operating hoist (*Fig.* 4).

When in position the blocks were adjusted to the correct line and camber, and the gaps between them packed with hand-rammed mortar. Short lengths of rubber tube with high-pressure hose-leads were passed into each cable duct to cover the joint and inflated to prevent the escape of mortar from the joints into the ducts. On completion of a joint the tube was deflated and drawn forward to the next joint, where it was re-inflated. A cable-sock and wire lead were used to pull the cables into the ducts. Cubes of the mortar had a compressive strength exceeding 6000 lb. per square inch at three days, and tensioning of the cables was then commenced.

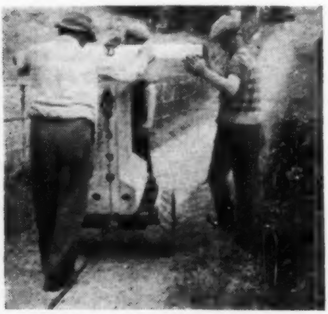


Fig. 3.—Transporting Blocks.

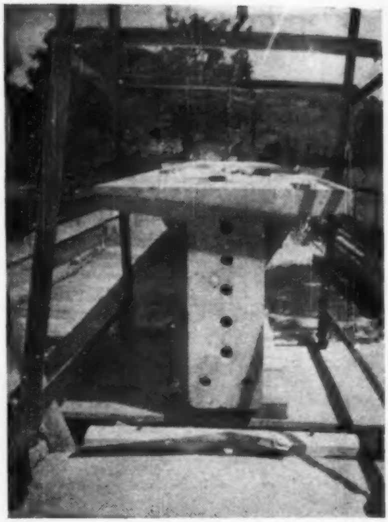


Fig. 4.—Gantry used for Placing Blocks.

Upon completion of the tensioning the beam had lifted 2 in. off the scaffold at the middle of the span. Grouting of the cable ducts was carried out with two pumps, using neat cement with a water-cement ratio of 0·45 and a plasticiser. The grout was injected from one end of the beam, since the cables had little curvature. Intermediate grouting points were provided but they proved to be unnecessary and were plugged as grouting proceeded.

Design.

The bridge is designed as a freely-supported beam to carry an imposed vertical load of 400 lb. per linear foot and to resist the horizontal force due to a wind of 60 miles per hour. Assuming an effective span of 108 ft., the greatest bending moments are 10,375,000 in.-lb. due to the dead load of 593 lb. per linear foot and 6,998,000 in.-lb. due to the imposed load. The properties of the tee-beam are given in *Fig.* 5. The seven cables, which have an eccentricity of 20·4 in. at midspan, comprise 84 wires of 0·276 in. diameter. The nominal initial tensioning force was 9000 lb. on each wire at midspan, and after losses (assumed to be 15 per cent.) the final force would

be 7600 lb. in each wire. The calculated basic stresses (lb. per square inch) are:

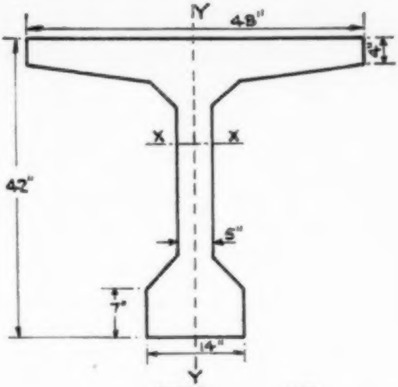
	Top flange	Bottom flange
Dead load . . .	+ 1420	− 2710
Live load. . . .	+ 930	− 1640
Wind on unloaded bridge	± 430	± 120
Wind on loaded bridge	± 1100	± 320
Initial prestress . .	− 620	+ 5500
Final prestress . .	− 530	+ 4680

The resultant stresses (lb. per square inch) are:

	Top flange	Bottom flange
At transfer . . .	+ 800	+ 2790
Bridge without load	+ 890	+ 1970
With wind on unloaded bridge . .	+ 1320 or + 460	+ 2090 or + 1850
Bridge with load .	+ 1820	+ 330
With wind on loaded bridge	+ 2920 or + 720	+ 650 or + 10

The total force in the wires at failure would be 84 × 13,400 lb. = 1,125,600 lb. The lever-arm is 14·4 + 20·4 − 2·7 = 32·1 in. The moment of resistance at near-failure is therefore 1,125,600 × 32·1 = 36,130,000 in.-lb. With a factor of safety of 1·5 for the dead load and 2·5 for the live load, this moment of resistance should be not less than 33,058,000 in.-lb.

Each wire was tensioned simultaneously from both ends and, allowing for friction in the duct, if the tension at midspan were 9000 lb. per wire, the tension at the face of the jack would be about 9500 lb. A $\frac{1}{8}$ in. take-up of the anchor-grip upon relaxation of the jack would be equivalent to a force of 300 lb. per wire, and if 500 lb. be allowed to offset the friction in the jack and anchorage, the jacking force as recorded on the pressure gauge should have been 10,300 lb. The calculated extension was 3·58 in., and, allowing $\frac{1}{4}$-in. take-up in the temporary and permanent grips, the reading on the scale of the jack should be $1\frac{13}{16}$ in.



	GROSS	NET
A	526 in.²	504 in.²
$\bar{y}$	15·2 in.	14·4 in.
I_{XX}	114129 in.⁴	104952 in.⁴
Z_T	7508 in.³	7288 in.³
Z_B	4259 in.³	3802 in.³
I_{YY}		45622 in.⁴
Z (top flange)		1901 in.³
Z (bottom flange)		6517 in.³

Fig. 5.—Properties of Tee-beam.

The specified crushing strength of works-cubes was not less than 7000 lb. per square inch at 28 days, or 5500 lb. per square inch at seven days, which was the minimum age at which the prestressing force was likely to be transferred to the concrete.

The cost of the work was £2550 including the temporary provision for pedestrian traffic, demolition of the old bridge, special access facilities, flood-relief channels, and reinstatement of the approach footpath. The bridge was designed

Fig. 6.—Bridge at Caterham.

by Udalls Prestressed Concrete, Ltd., and the tensioning of the cables was by the Gifford-Udall system. The precast blocks were made by Reconstruction Supplies, Ltd. The structure was constructed by direct labour by the Pembrokeshire County Council under the control of the County Engineer and Surveyor, Lt.-Col. W. Brundan, O.B.E. *Figs.* 3 and 4 are from photographs by Mr. W. E. Davies, bridge assistant to the County Engineer.

Bridge in Surrey.

Figs. 6 and 7 show a precast prestressed footbridge recently erected over the railway at Caterham, Surrey. The spans were precast on the site by the Gifford-Udall-CCL system with post-tensioned wires. The piers are of precast reinforced concrete and rest on reinforced concrete bases cast in place; the piers were erected in one night. The 32-ft. span over the track was assembled complete beside the railway, and was also erected in one night. The structural details were designed by Mr. E. W. H. Gifford for the Caterham and Warlingham U.D.C., of which Mr. L. V. Gordon, M.B.E., is the Engineer. The bridge was constructed by Messrs. Reed & Mallik, Ltd., and was completed in fourteen weeks.

Fig. 7.—View of Underside.

Congress of Precast Concrete.

The third International Congress of Precast Concrete will be held in Stockholm in June 1960.

The following subjects have been proposed for discussion by five countries: The application of vibration to precast concrete (United Kingdom); Shrinkage of concrete (Germany); Large precast concrete elements for buildings (Scandinavia); Industrial relations and productivity (France); Application of plastics to concrete (Spain).

Those wishing to contribute papers on these subjects should communicate with one of the following associations: British Cast Concrete Federation, 105 Uxbridge Road, London, W.5, England; Bundesverband der Betonsteinindustrie, Hausdorfstrasse 191, Bonn, German Federal Republic; Dansk Betonvare Industrie, Rosenorns Alle 18, Copenhagen, Denmark; Federation National des Fabricants de Produits en Beton, 11 Rue Alfred-Roll, Paris 17, France; Associacion Technica de Derivados del Cemento, Balmes 163, Barcelona, Spain. The latest date for receipt of contributions is 1 June 1959.

It is expected that the Swedish Precast Concrete Federation will organise an architectural competition for buildings recently erected in which precast concrete elements are used.

National Lending Library for Science and Technology.

The Department of Scientific and Industrial Research is to use part of the former Royal Ordnance Factory at Thorp Arch, near Boston Spa, Yorks, as a national lending library for science and technology. This will replace the lending library of the Science Museum Library, and is expected to become the most comprehensive library of its kind in the United Kingdom. The library is expected to be in operation in the year 1961, and will be available to research, industrial, educational, and other organisations.

Frames with Curved Beams.

By **THOMAS A. BARTA, B.Sc.**

POLYTECHNIC INSTITUTE OF TIMISOARA, ROUMANIA.

IN this journal for November 1957, Mr. A. Chronowicz showed that frames with curved beams can conveniently be analysed by the method of moment-distribution, using numerical integration based on the column analogy to determine the constants. The writer derived the same theory in a paper published in Roumania[1] and obtained simple formulæ not only for parabolic beams but also for circular beams which frequently occur as stiffening members of shell roofs.

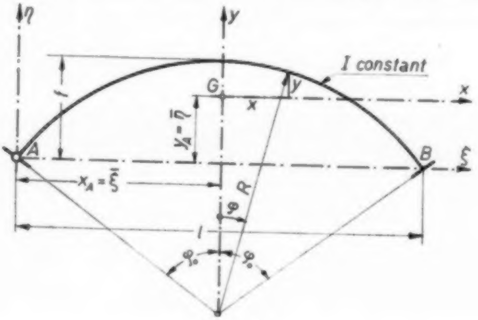


Fig. 1.

THE COLUMN-ANALOGY CONSTANTS.—With the notation given in *Fig.* 1, for a circular arch with constant moment of inertia,

$$l = 2R \sin \phi_0. \quad f = R(1 - \cos \phi_0). \quad x = R \sin \phi. \quad y = R\left(\cos \phi - \frac{\sin \phi_0}{\phi_0}\right).$$

$$\bar{\eta} = \frac{\int_s \eta \cdot \frac{ds}{EI}}{\int_s \frac{ds}{EI}} = R\frac{\int_0^{\phi_0} (\cos \phi - \cos \phi_0) d\phi}{\int_0^{\phi_0} d\phi} = R\left(\frac{\sin \phi_0}{\phi_0} - \cos \phi_0\right).$$

$$A = \int_s \frac{ds}{EI} = \frac{R}{EI}\int_{-\phi_0}^{\phi_0} d\phi = \frac{2R\phi_0}{EI}.$$

$$I_x = \int_s \frac{y^2\, ds}{EI} = \frac{R^3}{EI}\int_{-\phi_0}^{\phi_0}\left(\cos \phi - \frac{\sin \phi_0}{\phi_0}\right)^2 d\phi = \frac{R^3}{EI}\left(\sin \phi_0 \cos \phi_0 - 2\frac{\sin^2 \phi_0}{\phi_0} + \phi_0\right).$$

$$I_y = \int_s \frac{x^2\, ds}{EI} = \frac{R^3}{EI}\int_{-\phi_0}^{\phi_0} \sin^2\phi \, d\phi = \frac{R^3}{EI}(\phi_0 - \tfrac{1}{2} \sin 2\phi_0).$$

These formulæ are complicated, and very sensitive to error, particularly for small values of ϕ_0; they therefore involve the use of a calculating machine. This can be avoided by developing the formulæ as power series; for practical purposes it is sufficient to include only the first terms of the series.

The dimensions of the curved beam may be considered as functions of the radius and the angle ϕ_0, or as functions of the span-rise ratio and the angle ϕ_0. In the latter case the constants for a parabolic beam with variable moment of inertia $I = I_0 \sec \phi$ are obtained when $\phi_0 = 0$; a circular beam can therefore be

analysed by using the formulæ for a parabola multiplied by a correcting factor of the form $(1 + R\phi_0^2)$. Hence

$$\bar{\eta} = \frac{R\phi_0^2}{3}\left(1 - \frac{\phi_0^2}{10}\right) = \frac{2}{3}f\left(1 - \frac{\phi_0^2}{60}\right). \qquad A = \frac{2R\phi_0}{EI} = \frac{l}{EI}\left(1 + \frac{\phi_0^2}{6}\right).$$

$$I_x = \frac{2R^3\phi_0^5}{45EI}\left(1 - \frac{\phi_0^2}{7}\right) = \frac{4f^2l}{45EI}\left(1 + \frac{4\phi_0^2}{21}\right).$$

$$I_y = \frac{2R^3\phi_0^3}{3EI}\left(1 - \frac{\phi_0^2}{5}\right) = \frac{l^3}{12EI}\left(1 + \frac{3\phi_0^2}{10}\right).$$

STIFFNESS AND CARRY-OVER FACTOR.—The stiffness S and carry-over factor r for a symmetrical curved beam are given by

$$S = \frac{1}{4E}\left(\frac{x_A^2}{I_y} + \frac{1}{A} + \frac{y_A^2}{I_x}\right); \qquad r = \frac{\dfrac{x_A^2}{I_y} - \left(\dfrac{1}{A} + \dfrac{y_A^2}{I_x}\right)}{\dfrac{x_A^2}{I_y} + \left(\dfrac{1}{A} + \dfrac{y_A^2}{I_x}\right)}.$$

Substituting the column-analogy constants,

$$S = \frac{9}{4}\frac{I}{l}\left(1 - \frac{17}{70}\phi_0^2\right) \quad \text{and} \quad r = -\tfrac{1}{3}\left(1 + \frac{4}{35}\phi_0^2\right).$$

MOMENT M_0 AT THE END OF A CURVED BEAM.—Mr. Chronowicz indicated the equations $VI_\eta + HI_{\xi\eta} = M_0A\bar{\xi}$ and $VI_{\xi\eta} + HI_\xi = M_0A\bar{\eta}$.

Writing the solution in explicit form,

$$H_M = M_0A\frac{I_{\xi\eta}\bar{\xi} - I_\eta\bar{\eta}}{I_{\xi\eta}^2 - I_\xi I_\eta} \quad \text{and} \quad V_M = M_0A\frac{I_{\xi\eta}\bar{\eta} - I_\xi\bar{\xi}}{I_{\xi\eta}^2 - I_\xi I_\eta}.$$

Transferred to centroidal co-ordinates and with the stiffness inserted, these equations become $H_M = \dfrac{M_0}{4ES}.\dfrac{y_A}{I_x}$ and $V_M = \dfrac{M_0}{4ES}\dfrac{x_A}{I_y}$.

Therefore, using the column-analogy constants,

$$H_M = \frac{5M_0}{6f}\left(1 - \frac{\phi_0^2}{28}\right) \quad \text{and} \quad V_M = \frac{2M_0}{3l}\left(1 - \frac{2\phi_0^2}{35}\right).$$

FIXED-END MOMENTS AND FORCES AT THE ENDS, DUE TO DISPLACEMENTS.—In a similar manner, the following equations are obtained.

$$M_{\Delta H} = -\Delta_H\frac{y_A}{I_x} = -\Delta_H\frac{15EI}{2fl}\left(1 - \frac{29}{140}\phi_0^2\right).$$

$$H_{\Delta H} = \frac{\Delta_H}{I_x} = \Delta_H\frac{45EI}{4f^2l}\left(1 - \frac{4}{21}\phi_0^2\right).$$

$$M_{\Delta V}^A = -M_{\Delta V}^B = \Delta_V\frac{x_A}{I_y} = \Delta_V\frac{6EI}{l^2}\left(1 - \frac{3}{10}\phi_0^2\right).$$

$$V_{\Delta V} = \pm\frac{\Delta_V}{I_y} = \pm\Delta_V\frac{12EI}{l^3}\left(1 - \frac{3\phi_0^2}{10}\right).$$

(1) T. Barta and I. Grüner. "Analysis of frames with curved beams by the moment distribution-method" (in Roumanian) "Buletinul Stiintific si Tehnic al Institutului Politehnic Timisoara". Tom 1 (15) Fasc. 2–1957.

A Covered Market at Plymouth.

AN UNUSUAL " SHELL " ROOF.

A COVERED market now being built at Plymouth has an unusual type of north light shell roof, the shape of which varies from symmetrical segments at the ends to normal north-lights at the centre. The roof covers an area of about 220 ft. by 150 ft. and is supported by frames at centres of 32 ft. There are no internal supports. A model of the structure is shown in *Figs.* 1 and 2, and the arrangement is shown in *Figs.* 3 and 4.

Ten precast stiffening beams are provided in each bay of the roof. These enable the panels of the " shells " to be cast separately; the shuttering for each panel is supported by the structural frames, and the order in which the panels are cast is arranged to ensure that the stresses due to bending in the " shell " are as small as possible. Other advantages of the method are stated to include simplicity of design, speed of construction, and a reduction in the quantities of shuttering and scaffolding.

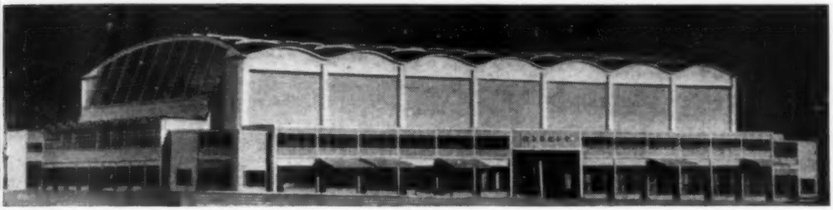


Fig. 1.

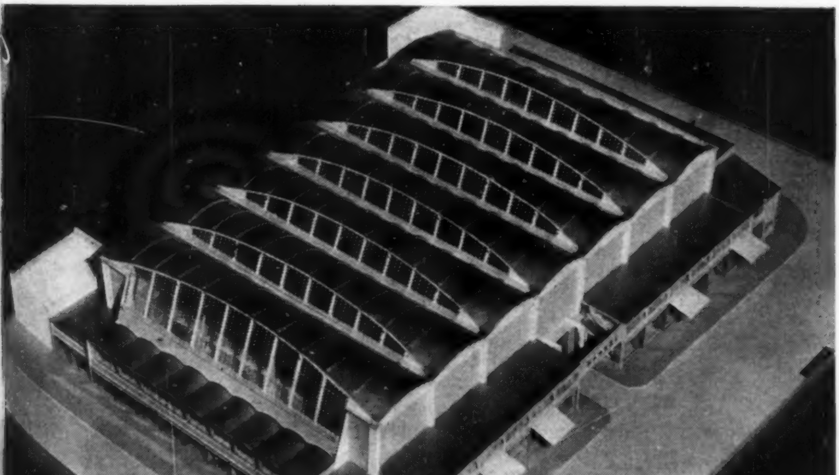


Fig. 2.

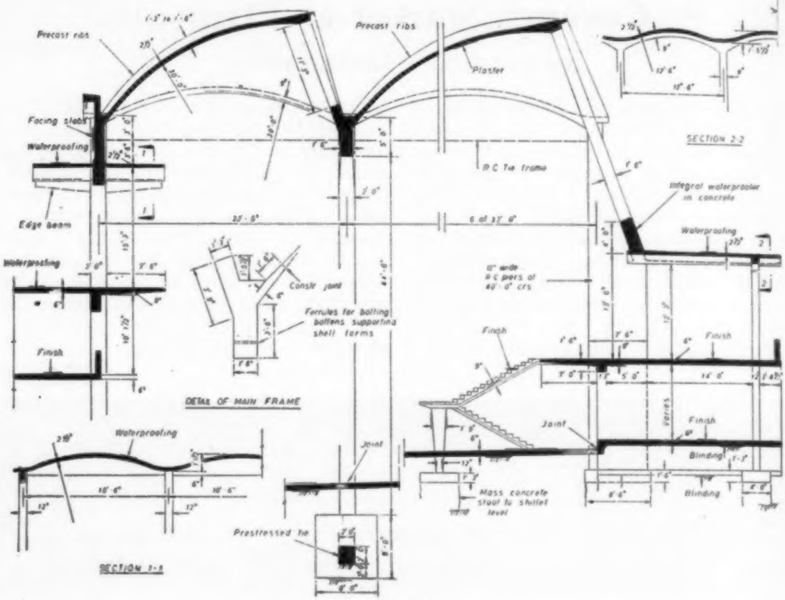

Precast ribs
Precast ribs
Plaster
Facing slabs
Waterproofing
Edge beam
R.C. Tie frame
Integral waterproofer in concrete
Waterproofing
SECTION 2-2
Waterproofing
Finish
Constr. joint
Ferrules for bolting battens supporting shell forms
DETAIL OF MAIN FRAME
10" wide R.C. piers at 40'-0" crs
Finish
Finish
Joint
Finish
Blinding
Blinding
Waterproofing
Joint
Mass concrete stool to stillet level
Prestressed tie
SECTION 1-1

Fig. 3.

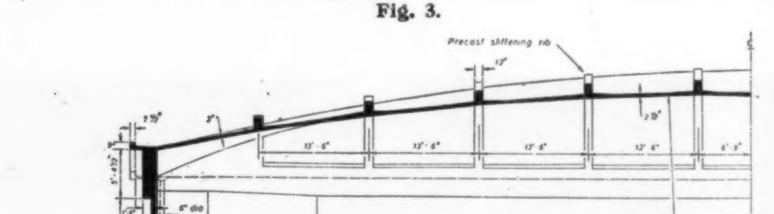

Precast stiffening rib
6" dia R.W. pipe
76'-0"
Expansion joint
Waterproofing
R = 307'-6"
Waterproofing
Cement fillet
Plaster
Constr. joint
Facing blocks
Expansion joint
DETAIL OF PRECAST RIB
Constr. joint
Finish
Non-retractable flat jack
SECTION 1-1
Finish
Joint
Blinding
Prestressed tie
Constr. joint
Building paper with sandwich of felt on blinding laid to smooth finish

Fig. 4.

Prestressed tie-beams are provided at foundation level between the columns of each frame. The first lift of each foundation block is cast to a depth of 2 ft. and covered with building paper and roofing felt, on which the tie-beams are cast. At this stage the tie-beams are free to slide on the foundation blocks. The beams are prestressed by post-tensioned cables, and the columns are then built on the ends of the beams. Freyssinet flat-jacks are placed between the columns and beams along one side of the building, and temporary hinges are provided along the other side. The frames are then completed and the flat-jacks expanded, the forces thereby exerted on the frames being such as to adjust the bending moments due to the weight of the structure to the most favourable values; the weight of the roof panels and the formwork is assumed to be 20 lb. per square foot of the plan area. The foundation blocks are completed (the feet of the columns are considered to be fixed) and the roof is then constructed.

The architects are Messrs. Walls & Pearn, and the contractors are Messrs. A. N. Coles (Contractors), Ltd. The structure was designed by the British Reinforced Concrete Engineering Co., Ltd., who are also supplying the reinforcement.

Shrinkage Stresses in Composite Floors.

MR. G. R. SMITH, of Prestressed Concrete Associates, London, writes as follows.—

SIR. The article on "Shrinkage Stresses in Composite Floors" by Mr. Bartlett in your number for December 1958, gives a method of determining stresses due to differential shrinkage in a composite beam which is similar to that given by Mr. F. Walley in "Prestressed Concrete Design and Construction". A more accurate assessment can be made by taking into account the reduction of the shrinkage force F due to the strain in the top of the precast beam. Shrinkage normally causes compressive stresses in the top of the precast beam which result in strains which relieve the shrinkage stresses. This reduction can be evaluated by equating strains.

Shrinkage force:

$$F = EA_x(s - s') \quad . \quad . \quad . \quad \text{(a)}$$

in which A_x is the area of the flange cast in place, s is the differential shrinkage strain between the flange and the precast beam (assuming that there is no connection), and s' is the final strain at the centre of the flange due to shrinkage acting on the composite member.

From formula (a),

$$s' = s - \frac{F}{EA_x} \quad . \quad . \quad \text{(b)}$$

Also,

$$s' = \frac{1}{E}\left(\frac{F}{A} + \frac{Fe}{Z_e}\right) \quad . \quad . \quad \text{(c)}$$

in which A is the total area of the composite member ($= A_x + A_y$), e is the distance between the centroid of the flange and the centroid of the composite member, and Z_e is the section modulus for the centroid of the flange.

From (b) and (c),

$$Es - \frac{F}{A_x} = F\left(\frac{1}{A} + \frac{e}{Z_e}\right)$$

Therefore F is

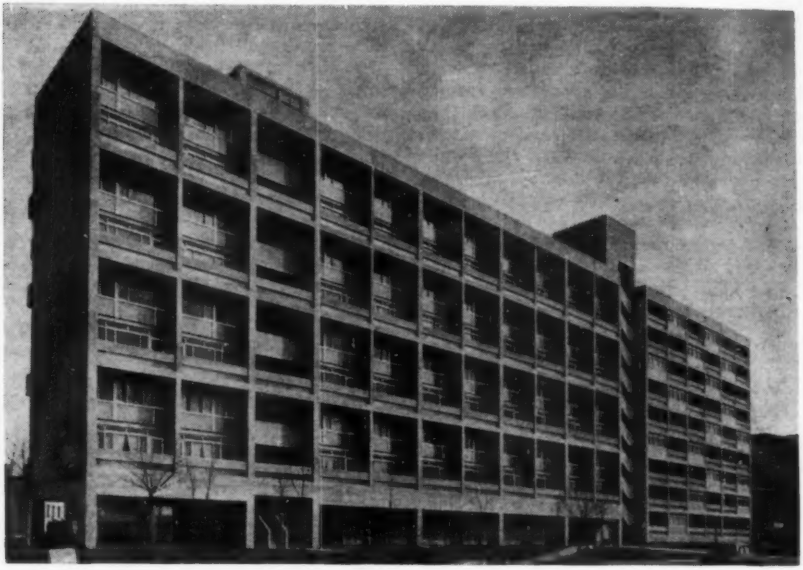
$$\frac{Es}{\left(\frac{1}{A} + \frac{1}{A_x} + \frac{e}{Z_e}\right)},$$

which should be compared with EsA_x as given by Mr. Bartlett. It can be shown that for ordinary members of tee-section this modification results in a reduction of the shrinkage force F of between 33 per cent. and 50 per cent.

Conference on Earthquake Engineering.

THE second World Conference on Earthquake Engineering, organised by the Science Council of Japan, is to be held in Tokyo and Kyoto from July 11 to 18, 1960. Further information may be had from Professor Kiyoshi Muto, Science Council of Japan, Ueno Park, Taito-Ku, Tokyo, Japan.

Flats Built in Sixteen Months.



THE illustration shows a block of flats recently built in reinforced concrete in Lambeth, London, in sixteen months; 3400 cu. yd. of concrete were placed in 120 working days. The structure is nine stories high, and comprises forty maisonettes each of 770 sq. ft., sixteen flats each of 850 sq. ft., twenty flats each of 410 sq. ft., and a caretaker's flat.

The foundations are of plain concrete. The main columns up to the lowest floor are at 29 ft. 6 in. centres, above which the frame consists of precast concrete units with the external face finished in bush-hammered granite concrete. The fronts of the balconies and the parapet walls were precast and bush-hammered on the site; it is believed that this is the first building in the London area on which this type of finish is used on such a large scale. Inflatable rubber tubes were used to form hollow cores in the precast balustrade units. The roof is finished with asphalt on an insulating layer of vermiculite, and the roof duct is incorporated in the fascia beam. All services, including rainwater pipes, are in ducts.

The architect was Mr. L. G. Creed, the consulting engineers Messrs. F. J. Samuely & Partners, and the contractors Messrs. Wates, Ltd. The cost of the structure was £250,000.

Congress on Building Research.

THE International Council for Building Research, Studies, and Documentation is arranging an international congress at Rotterdam from September 21 to 25 next. Among the papers to be read and discussed are "Design and Calculation of Structures and Safety Factors", by Professor E. Torroja (Italy); "Research Problems Relating to the Application of Heavy Concrete Elements", by Professor G. Kutznetsow (U.S.S.R.) and Dr. M. Jacobsson (Sweden); and "Fundamental Aspects of the Transmission of Knowledge", by L. M. Giertz (Sweden). Further information can be obtained from the Secretariat of the Council, P.O. Box 299, Rotterdam, The Netherlands.

The Design of Hinges.

A RECENTLY-PUBLISHED booklet* gives a report on tests of concrete hinges for use in buildings and bridges. The laboratory specimens included full-size concrete hinges and models made of artificial resin tested photo-elastically in polarised light in order to assess the distribution of stresses. As a result of the investigations the following principles are recommended for the design of concrete hinges.

1.—The effective width of the hinge should be one-third of the total width of the member. The concrete should have a cube strength at 28 days of not less than 2280 lb. per square inch. The size of the aggregate should not exceed 7 mm.

2.—The cross-sectional area A_s of the reinforcement should be calculated from $P_{\text{ultimate}} = cA_c + tA_s$, in which c is the cube strength of the concrete and t is the yield-point of the steel. The permissible load is one-third of the ultimate load.

3.—Of the reinforcement, 40 per cent. should be arranged so that the bars cross

* Deutscher Ausschuss für Stahlbeton. Issue 125. Federgelenke. By Karl Kammüller and Ottfried Jeske. (Berlin: Wilhelm Ernst & Sohn. Price 10 D.M.)

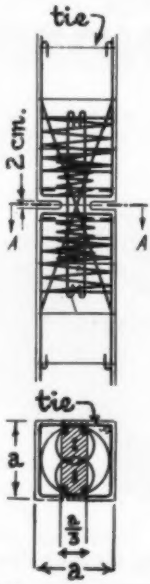


Fig. 1.—Reinforcement at Hinge (Four inner and four outer helicals).

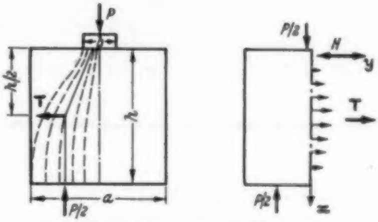


Fig. 2.—Transverse Tension.

at the midpoint of the hinge, and the remaining 60 per cent. should be placed longitudinally as in *Fig.* 1. The diagonal bars should be continued into the column at a slope of not less than 2 in 1.

4.—The bars in the column should be bent at an angle of 90 deg. parallel to the hinges, to which they should be placed as closely as possible.

5.—Where angular rotation is likely to exceed 0·01, dynamic tests are advisable.

6.—In buildings, the joint should be rounded with a radius r of $\frac{a}{24}$. A decrease in curvature results in an increase of concentration of stresses at the hinge.

7.—In bridges the joint should be formed by a hyperbola in order to reduce transverse tensile stresses, using the equation

$$\left(\frac{y}{4r}\right)^2 - \frac{1}{4}\left(\frac{x}{r}\right)^2 = 1.$$

The slope of the asymptote should be one in two, and the total width of the joint at the edge should be

$$2x = 8r\sqrt{2}.$$

8.—Transverse reinforcement should be designed for $T = \frac{P(a-b)}{4h} = \frac{P}{4}$ (*Fig.* 2).

In buildings where the forces involved are small, the reinforcement should consist of ties spaced from 4 in. to 8 in. apart and concentrated at $\frac{x}{a} = 0{\cdot}2$ (according to Bortsch).

For bridges, and for buildings where forces are large, helical binding should be used as shown in *Fig.* 1. The diameter of the inner helices should exceed the width of the hinge by $1\frac{1}{2}$ in. The height of the

binders should be $\frac{x}{a} = 0{\cdot}6$ for the inner helices and $1{\cdot}2$ for the outer helices. The pitch of the bindings should be one-fifth of the diameter of the hinge.

The distribution of the total tension T may be assumed to be less than $0{\cdot}20T$ for the inner helices and greater than $0{\cdot}12T$ for the outer helices. The cross-sectional area of each helix for ring tension R should be determined for a permissible tensile stress of 14,200 lb. per square inch.

Concrete Pipes in the U.S.S.R.

In an article in "Gidrotekhn Stroit", Mr. K. M. Khuberyan describes the following methods used in the U.S.S.R. for reducing bending moments in large concrete pipes.

Fig. 1 shows the cross section of a water conduit for a hydro-electric station. The internal profile is formed by two semicircles connected by short straight lengths, and the bottom of the pipe forms a thicker flat base. By this means bending moments due to external loads are counteracted by those due to internal pressures (which are not present in a circular pipe), with a saving in this case of about 13 per cent. of the cost of a circular pipe. *Fig.* 2 shows a cross section of a similar pipe, the cost of which was about 25 per cent. less than that of a circular pipe.

An alternative method of introducing bending moments which oppose those due to external loads is shown in *Fig.* 3, which represents the reinforcement of a feed-syphon of about 8 ft. diameter, now being built in the Donetz region. In this method the steel is prestressed by means of diametral tie-rods and hydraulic jacks

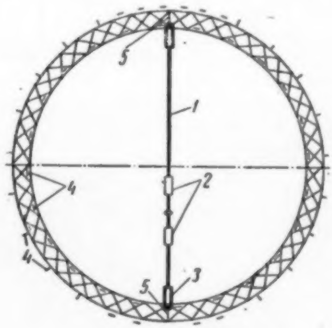


Fig. 3.

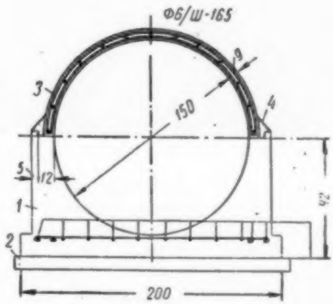


Fig. 4.

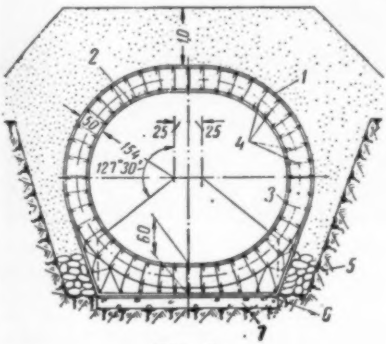


Fig. 1.

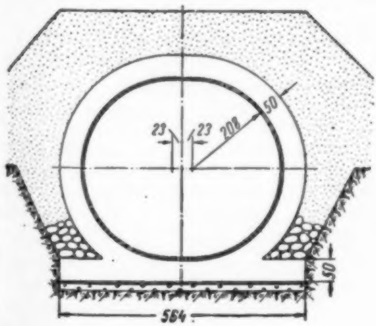


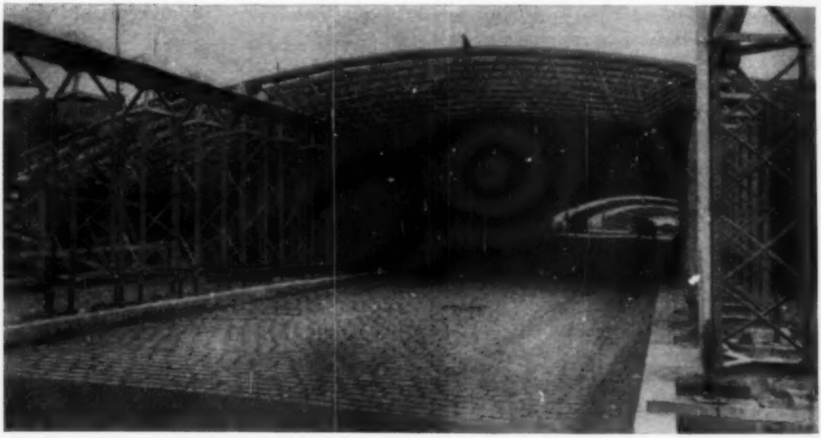
Fig. 2.

Fig. 5.

before the concrete is placed. When the concrete has hardened the tie-rods are released and the concrete is prestressed by the steel which produces bending moments of opposite sign to those due to external loads. It is stated that by this means the thickness of the pipe was reduced from 10 in. to 6½ in.; its length is about 3¾ miles.

A pipe in which there is no internal pressure-head is shown in *Fig.* 4; the upper half forms a two-hinged arch and the lower half forms a thick base. *Fig.* 5 shows a pipe with an elliptical internal profile. The author states that the cost of forming such special sections, which approximate as closely as possible to the pressure line of the loads, is appreciably less than the cost of the materials saved.

Supports for Heavy Shuttering.



A GIRDER and trestle which have recently been developed for supporting heavy shuttering are shown above. The girder, known as the "D.S.L. 500", consists of intermediate sections, end sections, bearer pins, and turnbuckles. The end sections are adjustable in length and by combining various intermediate sections any span from 9 ft. 10 in. to 91 ft. can be obtained. The upper booms of the sections are connected by pins and the lower booms by turnbuckles, which can be adjusted to provide various curvatures. For spans of up to 35 ft. single sections would be generally used, and for longer spans double sections, as in the illustration, are recommended. For heavy loads and very long spans a girder with bracing units is available. For example, a single girder can support a load of 10 tons over a span of 30 ft., a double girder a load of 14½ tons over a span of 49 ft., and a braced girder a load of 25 tons over 72 ft.

The trestle, known as the "D.S.L. Trishore", consists of various intermediate sections, an adjustable base, and a pyramidal or flat head. Any desired height may be obtained; the sections are bolted to the adjacent units and can be folded flat for ease of storage and transit. The trestle can support a load of 30 tons. The system is supplied by Rapid Metal Developments, Ltd.

The Cost of Placing Stiff Concrete.

THE following is an abstract of a report of the Department of Scientific and Industrial Research describing an investigation into the extra cost of placing stiff concrete and is given by permission of the Director of Building Research ; Crown copyright is reserved. The report describes the difficulties and expense that result if concrete mixtures are not workable enough to be compacted with the means available. To ascertain the effect of consistence on the cost of producing concrete, studies have been made on sites where nominal mixtures were used and also on sites where mixtures specially produced to a laboratory specification were used and where savings of cement were claimed largely as a result of the use of drier and leaner mixtures. The purpose of the studies was to compare the savings of cement with any additional costs of producing, placing, and compacting the drier mixtures.

Site A.

In the case of a building in London 1000 cu. yd. of concrete were specified to have minimum compressive strengths of 3000 lb. and 4000 lb. per square inch 28 days after compaction by vibration. Compared with a nominal mixture this concrete was claimed to save 11·5 per cent. of the cement content, valued at £165, or about 3*s.* per cubic yard. The costs are given in *Table* I, which includes

TABLE I.

COST OF 1000 CU. YD. OF CONCRETE.

Cement	£1500
Aggregates	£1550
Mechanical plant (for concreting only)	£2630
Mechanical plant (for other purposes)	£270
Static plant (scaffold, props, etc.)	£1140
Plywood for shuttering	£745
Softwood for shuttering	£1750
Carpenters	£4540
Labourers (time on concreting)	£2190
	£16,315

cement at £6 per ton, aggregates at £1 per cubic yard, plant at weekly hire rates, labour at 6*s.* 6*d.* per hour for tradesmen and 6*s.* per hour for labourers. The cost of reinforcement, fuel, and overhead expenses is not included. If the average saving of cement claimed of 3*s.* per cubic yard be compared with the cost of the concrete of about 320*s.* per cubic yard, it is seen that small effects on the rate of production resulting from the use of stiff concrete can easily outweigh the saving of cement.

As on other sites observed where stiff concrete was used it was not possible, except in the case of slabs, to place the concrete directly from the skip into the shutters. The practice was to deposit the concrete on a banker, take it to the shutters in wheelbarrows or dumpers, and shovel it into place, with consequent increase in labour costs. The use of a skip had to be abandoned for some of the work because the concrete would not leave the hopper by gravity but had to be shovelled out, and much of the concrete was lifted by pail and rope.

Because of the discontinuous nature of the concreting it was not easy to determine the cost of the extra labour employed, but the saving of only one man during the period of the concrete work would be equivalent to £500, compared with the amount of cement saved valued at £165. Also, the cost of compaction by immersion vibrators was equivalent to a plant-hire charge of £370. The appearance of the finished concrete was no better than on sites on which nominal mixtures were used.

Site B.

Another study was made during the construction of an eight-story building where a stiff mixture was used. The standard of workmanship and organisation was high. The structure contained about 8000 cu. yd. of concrete and the saving of cement claimed was 13 per cent., valued at about £1600. The building comprised reinforced concrete flat slabs supported on columns and walls. Apart from the gable walls and some parts near the staircases and lift-shaft, there was little to prevent easy concreting. Studies were made of work on the fifth, sixth, and seventh floors when the best rate of production should have been achieved. The concrete was produced by a 21/14 mixer, and hoisted in a side-discharge skip by a crane.

The concrete was too stiff to flow from the skip into the wall and column shutters, and in these cases it was deposited on a banker and shovelled into place. The workability, as indicated by the discharge from the skip, varied somewhat. Some assistance was always required, and at times the concrete could be extracted only by a man standing in the skip and treading it out; at other times an immersion vibrator was used for the purpose.

The time taken and the number of men required to place the concrete in the column and wall shutters were particularly affected by the stiffness of the concrete; slowness in placing also reduced the output of the men working around the mixer. The concreting of floor slabs progressed as fast as the crane would allow, apart from the delay in emptying the skip. However, compared with similar work on other sites, additional labour was required to spread and compact the concrete, particularly when only a spaded finish was achieved. Compaction of the slabs by immersion vibrators appeared somewhat haphazard and in the case of slabs full compaction was difficult to verify.

One complete floor was constructed in about fourteen working days and comprised about 280 cu. yd. of concrete. The labour employed to mix and place the concrete was as follows in man-hours per cubic yard: slabs, 2·6; walls, 3·6; columns, 7·4; stairs and beams, 3·8.

A study was made of the output of three men placing and compacting concrete in a wall at a rate of 5·4 cu. yd. in 140 minutes. Two men received the concrete from the skip and shovelled it into the shutter and the third compacted it with an immersion vibrator. Ignoring avoidable idle time, the men's time was spent as follows: Vibrating—Preparing equipment, 4 per cent.; vibrating concrete, 87 per cent.; waiting, 9 per cent. Placing concrete—Preparing equipment, 6 per cent.; emptying skip, 24 per cent.; shovelling and placing, 48 per cent.; waiting, 22 per cent.

In this case the saving in cement was roughly equivalent to the cost of one labourer during the period of 14 days, and this was more than offset by the cost of the extra labour required compared with the quicker placing possible with a wetter mixture, apart from the effect of the slow rate of placing on the rate of production of the other men and plant on the site. More limited studies on other sites have confirmed in general the data obtained on sites A and B.

Site C.

This building was a multiple-story office building costing £2,500,000 and of similar construction to example B. The structural design and detailing of the reinforced concrete work were arranged so as give as little hindrance to construction as the architectural requirements would allow. More than 30,000 cu. yd. of concrete were used in nominal mixtures of generally 1 : 2 : 4 and 1 : 3 : 6 for foundations, 1 : 1½ : 3 for most of the superstructure, and 1 : 1 : 2 for columns. For the superstructure, the materials were proportioned in a weigh-batcher and the results appeared to be at least as accurate as on sites where more rigorous specifications were used. The consistency of the concrete was such that the concrete flowed into place with little manual assistance, and it was easy to regulate the flow from the crane-skip to deposit the correct quantity directly into the shutters. The 1 : 1½ : 3 and 1 : 1 : 2 mixtures flowed readily without segregation. A vibrator was available and was used to assist the concrete to flow into angles and around congested reinforcement rather than to compact it.

Tower cranes of 5 tons capacity were used for hoisting concrete. Each floor was constructed, on average, in seven working days and included 520 cu. yd. of concrete. The rate of placing the concrete was generally limited only by the speed of the crane. The time required to empty the 1-cu. yd. side-discharge skip seldom exceeded a few seconds.

Slabs, walls, and the larger columns were cast at the rate of 14 cu. yd. per hour, or about 0·6 man-hour per cubic yard. In the case of the 9-in. by 9-in. columns the total time spent was 6 minutes per column, or 4 man-hours per cubic yard.

The estimated cost of site overheads (say 8 per cent. of labour costs), the plant, and the proportion of labour such as plant operators, maintenance men, etc., who were on the site whatever the rate of production, was about £1600 per week averaged over the whole contract time of three years. The cement used is

estimated at 10,000 tons, valued at, say, £60,000. If stiffer mixtures had yielded a saving of cement of 10 per cent., or £6000, this would have been lost if its use had extended the work by as little as three weeks in respect of these items alone, and the effect on the work as a whole might have been even greater.

CONCLUSIONS.—On many sites the cost of plant and labour is more important than the cost of cement, and on others the rate of concreting has a great effect on the work as a whole, so that saving of cement by the use of stiff concrete without considering the cost of placing it is unlikely to effect any saving in total cost. Where stiff concrete does not affect the rate of placing, or where the concreting operation has little effect on the work as a whole, it should be possible to make economies by the use of stiff concrete.

[This report is referred to in the Editorial Note on p. 103 of this number.]

FIFTY YEARS AGO.

From "CONCRETE AND CONSTRUCTIONAL ENGINEERING", March, 1909.*

THE ANGEL ROAD BRIDGE.—This bridge, which was erected for the Great Eastern Railway Co., carries the public road over the railway [in North London] in place of a level crossing. This is one of the largest bridges and approaches in reinforced concrete in the United Kingdom.

The design is on the Considère system, and consists of 17 equal spans, three of which are on a curve, designed as continuous girders, 42 ft. 9 in. between the centres of bearing and 5 ft. 4½ in. apart, carrying a roadway 24 ft. wide, together with 2 ft. of footway on both sides, the remaining 6 ft. of the latter, together with the parapets, being supported by cantilevers.

The floor is 6 in. thick, weighing, together with the road metalling, 0·77 ton per super yard, and carries a load of 6½ tons on a wheel 20 in. wide; this weight is assumed to be transmitted through the road metal at an angle of 1 in 1 and into the body of the flooring at an angle of 1½ in 1, the total distributed superload being therefore 3·2 tons per super yard.

[This bridge has been repaired during the years and is still in use.]

* "Concrete and Constructional Engineering" appeared in alternate months until September, 1909.

Failure of a Bridge during Construction.

AN inquiry into the collapse in June 1958 of a span of the Second Narrows bridge, at Vancouver, B.C., Canada, indicates that the cause was the failure of the grillage supporting the falsework due to an error in the calculations.

The bridge comprises spans varying in length from 1100 ft. to 120 ft., in steel and prestressed concrete, which were cantilevered into position. At the junctions of the spans supports for the falsework were provided in the form of trestles carried on steel grillages on piles driven into the bed of the river. One of these spans collapsed, with the loss of eighteen lives, as it was being placed in position and nearing its second temporary supporting trestle. It appears that "the entire area of the steel beams in the grillage had been used in the calculations, whereas only the web area should have been considered as effective. As a result the design of the grillage was based on a shear stress about 50 per cent. too small. Also, the calculation for stiffeners in the grillage was based on the 1-in. flange thickness instead of the 0·653 in. web thickness." There were indications on the drawings that these mistakes had been noticed, but they had not been corrected.

Training Courses in Concrete.

THE programme of the training courses to be held by the Cement and Concrete Association during the year 1959 is now available. As in previous years the courses will be held at the Association's centre at Wexham Springs, Buckinghamshire, where hostel accommodation is available for those attending the courses. The courses include the following:

March and April: Structural concrete (for supervisors).

May and June: Structural concrete (for engineers).

June: Soil-cement roads and airfields (for engineers).

June: Concrete road and soil-cement construction (for supervisors).

June and July: Course for architects.

August and September: Concrete road construction (for engineers).

September and October: Properties of concrete, proportioning concrete, and control of quality (for engineers).

November: Concrete products (for members and employees of firms subscribing to the Research Committee for the Cast Stone and Cast Concrete Products Industry).

November: Prestressed and reinforced bridges (for engineers).

Full details may be had from the Association at 52 Grosvenor Gardens, London, S.W.1.

Lectures on Building.

THE following lectures have been arranged by the Ministry of Works. Admission is free.

Thermal Insulation of Buildings, by P. A. Denison. Percival Whitley College, Francis Street, Halifax. March 18. 7.15 p.m.

Good Practice in Domestic Drainage, by F. J. Crabb. N.E. Essex Technical College, Sheepen Road, Colchester. April 2. 7.15 p.m.

Safety in the Building Industry, by J. A. Hayward. Cornwall Technical College, Trevenson, Pool, Redruth. March 17. 7 p.m. And College of Technology, Byrom Street, Liverpool. March 23. 7.15 p.m.

Work Study in the Building Industry, by C. A. Francis. S.W. Essex Technical College, Forest Road, London, E.17. March 16. 7.15 p.m.

Problems of Plastering and Rendering, by E. L. Westbrook. College of Further Education, Avenue Road, Grantham. March 17. 7.15 p.m.

Plastics in the Building Industry, by H. N. Davies. Technical College, Wrexham. March 17. 7 p.m. Also Shire Hall, Llangefni. March 18. 7.15 p.m. And Technical College, Connah's Quay. March 19. 7 p.m.

Structural Fire Protection, by G. J. Langdon-Thomas. Mining & Technical College, Library Street, Wigan. March 19. 7.30 p.m.

Five Centuries of Building in Halifax and District, by C. Sunderland. Percival Whitley College, Francis Street, Halifax. March 20. 7.15 p.m.

Soil Mechanics in the Building Industry, by A. F. Mendoza. Technical College, The Park, Cheltenham. April 3. 7 p.m.

ETE

and

(for
sub-
ee for
oducts

forced

n the
rdens,

anged
ion is

s, by
ollege,
7.15

inage,
hnical
hester

y, by
hnical
druth
ge of
rpool

ustry
hnical
E.17

ering
arther
tham

y, by
Wrex-
Shire
p.m.
Quay

G. J.
hnical
March

alifax
rcival
lifax

ustry,
ollege,
p.m.